REMARKS

Claim Rejections

Claims 1, 3-4, 6-7, 9-15, and 17-24 are rejected under 35 U.S.C. 112, second paragraph, as being indefinite. Claims 1, 4, 6, 7, 9, 13-15, and 18-24 are rejected under 35 U.S.C. 102(e) as being anticipated by US Patent #6,865,791 to Cook et al. Claims 1, 9 and 10 are rejected under 35 U.S.C. 102(e) as being anticipated by US Patent #6,591,573 to Houghton. Claims 3, 12, and 17 stand rejected under 35 U.S.C. § 103(a), as being unpatentable over Cook et al.

Drawings

The Examiner has objected to the drawings under 37 C.F.R. § 1.83(a) insofar as the "spring isolator," referred to in Applicant's claim 12, was not illustrated in the figures. Since claim 12 has been deleted from Applicant's claims it is not believed that any drawing corrections are necessary.

Claim Amendments

By this Amendment, Applicant has canceled claims 12 and 17-19 and has amended claims 1, 4, 13, 20, 23, and 24 of this application to overcome the Examiner's objections thereto and rejections thereof. It is believed that the amended claims specifically set forth each element of Applicant's invention in full compliance with 35 U.S.C. § 112, and define subject matter that is patentably distinguishable over the cited prior art, taken individually or in combination.

Applicant respectfully submits that the objectives and resulting structures of the cited art to Cook et al. and Houghton are distinct from Applicant's recited structure. For instance, the objective of Cook et al. is to solve shear failure in a structure. Specifically, in Fig. 1b, by inserting a supporting ring 4 in a vertical hollow, member 3 can prevent a diagonal strut 2 from punching through. In comparison, the structure of the present invention is configured <u>to reduce the moment of the supported member at the support spot, and a bending moment value at the joint becoming uniform</u>.

The objective of Houghton is to prevent defects by welding the prior art - beam's flange directly to the column for resistance to earthquakes, explosions, and tornadoes. In other words, as shown in Fig. 3, the gusset plate 18 is attached to the flank of a beam 4, and the gusset plate 18 is welded to a column 2.

In comparison to the above, as disclosed in the background of the present invention, there are **two known** methods to solve the problem of defects. One is to enhance the intensity of the beam. However, this is ineffectual to solve the problem that stress concentrates at the joints of the members, thereby causing poor quality control of the welding job. The other method is to reduce the beam cross section, which will create the need for larger members.

Applicant teaches and recites a **third** novel solution to such defects. Namely, Applicant uses moment resisting connections (welding) to join the supporting member 29 to the connection element 26 at the joint in one end. The other end is contacted by the supported member at the support spot without welding. In order to assist the Examiner in further appreciating the differences between Applicant's invention and the cited art, Applicant further provides herewith Applicant's article, *Cyclic Testing of Steel Beam-to-Column Connections with Supporting Members.*

It is axiomatic in U.S. patent law that, in order for a reference to anticipate a claimed structure, it must clearly disclose each and every feature of the claimed structure. Applicant submits that it is abundantly clear, as discussed above, that Cook et al. do not disclose each and every feature of Applicant's amended claims and, therefore, could not possibly anticipate these claims under 35 U.S.C. § 102. Absent a specific showing of these features, Cook et al. cannot be said to anticipate any of Applicant's amended claims under 35 U.S.C. § 102.

Likewise, Applicant submits that it is abundantly clear, as discussed above, that Houghton does not disclose each and every feature of Applicant's amended claims and, therefore, could not possibly anticipate these claims under 35 U.S.C. § 102. Absent a specific showing of these features, Houghton cannot be said to anticipate any of Applicant's amended claims under 35 U.S.C. § 102.

It is further submitted that Cook et al. do not disclose, or suggest any modification of the specifically disclosed structures that would lead one having ordinary skill in the art to arrive at Applicant's claimed structure. Thus, it is not

believed that Cook et al. render obvious any of Applicant's claims under 35 U.S.C. § 103.

Summary

In view of the foregoing amendments and remarks, Applicant submits that this application is now in condition for allowance and such action is respectfully requested. Should any points remain in issue, which the Examiner feels could best be resolved by either a personal or a telephone interview, it is urged that Applicant's local attorney be contacted at the exchange listed below.

Respectfully submitted,

Date: June 5, 2008   By: ____________________

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CYCLIC TESTING OF STEEL BEAM-TO-COLUMN CONNECTIONS WITH SUPPORTING MEMBERS

Cheng-Chih CHEN[1], Jen-Fu LO[2], Yuh-Chyurn DING[3], Lo MAO[4], and Chi-Chun TAO[5]

SUMMARY

This paper presents an experimental investigation of moment connections supported by an additive member to move the maximum flexural moment of the beam away from the beam-column interface, and to reduce the moment demand at the beam-column interface. Theoretical study shows that the yielding mechanism of the beam depends on the elastic stiffness of the supporting member. The beam can form plastic hinge on the beam at the supporting location or develop shear yielding within the supporting span of the beam. Full-scale tests confirmed the behavior, and demonstrated that the proposed connections can diminish the potential brittle failure in the beam-column interface and develop satisfactory hysteresis behavior.

***Keywords**: beam-to-column connection, supporting member, plastic hinge, hysteresis.*

INTRODUCTION

Steel moment-resisting frames (MRFs) have been widely used to resist earthquake-induced forces because of their ductile behavior. MRFs are designed to remain elastic during small to medium earthquakes, but the frames are expected to develop inelastic, ductile behavior when subjected to a large earthquake. However, numerous beam-to-column moment connections in steel moment-resisting frames were damaged with limited inelastic behavior during the 1994 Northridge and 1995 Hyogoken-Nanbu earthquakes (Miller 1998; Nakashima et al. 1998). To improve the cyclic behavior of the moment connections, research conducted after the earthquakes emphasized on either reinforcing the beam-column interface or weakening the beam section, and both improvements led to the formation of the plastic hinge in the beam section away from the beam-column interface. Reinforced connections can be achieved by using cover plate, wing plate, vertical rib, and haunch (Engelhardt and Sabol 1998; Chen et al. 2004; Uang et al. 1998). Reduced beam section (RBS) has been verified to be an effective improvement for the moment connection and has been widely used in the US (Chen et al. 1996; Engelhardt et al. 1998).

Rather than forming the plastic hinge at the beam-column interface, the post-Northridge connections were improved to develop the plastic hinge on the beam section away from the interface to assure reliable, stable plastic deformation capacity. Nevertheless, for both RBS and reinforced connections, the beam-column interface still has the maximum bending moment when the moment-resisting frames are subjected to the lateral force exerted from the seismic excitation. In this paper, an intention to reduce the moment demand at the beam-column interface is proposed by adding supporting members to enhance the seismic performance of the moment connection. Based on the additional stiffness provided by the supporting member, the maximum bending moment may occur away from the beam-column interface. By varying the elastic stiffness provided by the supporting member, the beam may either develop flexural plastic strength or reach shear yielding capacity. Full-scale experiment was carried out to verify the cyclic behavior of the moment connections with the

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supporting member. Test results are discussed with emphases on the hysteresis behavior and mode of failure.

ROLE OF SUPPORTING MEMBERS

For a moment-resisting frame subjected to earthquake-induced moments, inflection points are, in general, assumed to occur at the mid-spans of the beams and mid-heights of the columns. For the bending moment distribution of a beam, maximum bending moments occur at both ends of the beam which are the beam-column joint. To join the beam and column, shear tabs and the beam flange complete joint penetration weld are usually designed. When a supporting member is added to provide a support in the beam, at a short distance from the column face, the bending moment at the joint will be reduced. Figure 1 presents the distribution of the bending moment and the shear force for a cantilever beam with a support having different spring stiffness. Due to the reaction of the support, the bending moment at the fixed end is varied.

Elastic spring stiffness K_s is assumed for the supporting member. Five cases indicated in Fig. 1 can be considered to elaborate the effect of the supporting member on the distributions of the bending moment and shear force. The distributions of the bending moment and shear force in the beam depend on the stiffness K_s provided by the supporting member and are discussed as follows.

For case (1), K_s=0, the cantilever beam has the maximum bending moment at the fixed end. In case (2), $0 < K_s < (2L_c^3/3LL_s^2)K_b$, maximum bending moment, with magnitude between PL and PL_c, is expected at the fixed end a, where $K_b = 3EI_b/L_c^3$ and I_b is the moment of inertia of the beam section. In case (3), $K_s = (2L_c^3/3LL_s^2)K_b$, constant maximum bending moment can be obtained within the supporting span L_s. Moreover, the shear force within the supporting span L_s is zero because of the pure bending moment within the span. Case (4), $(2L_c^3/3LL_s^2)K_b < K_s < \infty$, the bending moment within the supporting span L_s is less than that at the point b. Furthermore, double curvature is expected when the K_s is increased. In cased (5), $K_s = \infty$, the maximum bending moment at the beam occurs at point b and the bending moment at the fixed point a is $0.5PL$ only.

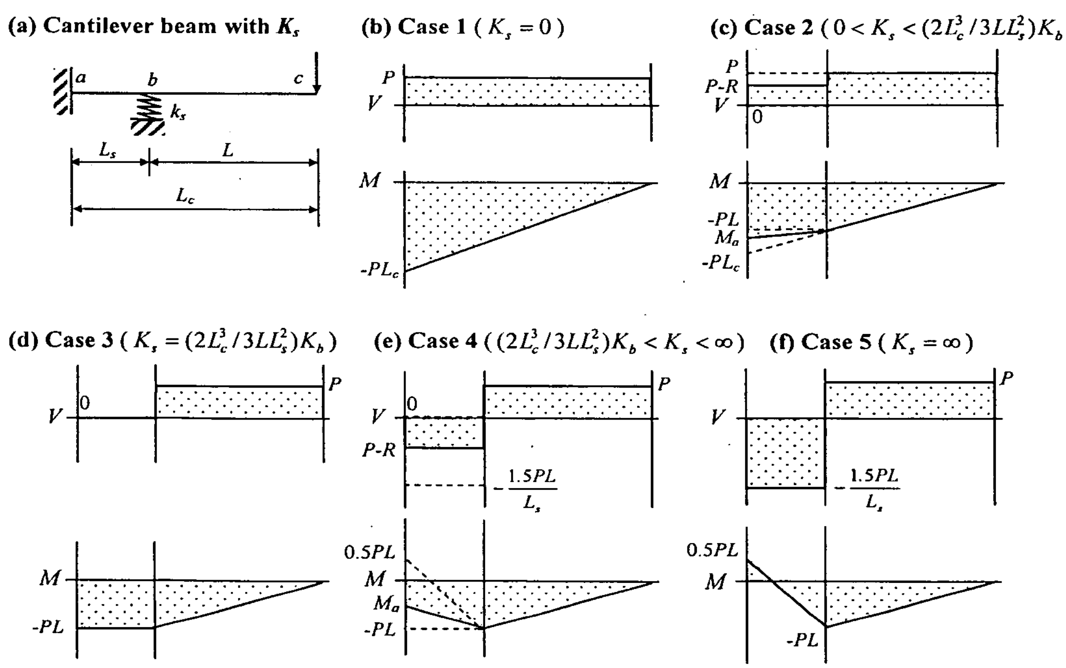


Fig. 1 Distributions of bending moment and shear force

The above mentioned behavior is based on the assumption of the beam remaining elastic. Nevertheless, the flexural stress on the beam cross section may reach the yield strength, and the shear stress may attain the yield shear stress. Owing to the inelastic behavior of the beam cross section, the flexural plastic strength M_P and shear yielding strength V_n should be taken into account for the distributions of the bending moment and shear force. Therefore, plastic hinge may be formed while the beam cross section developing its flexural plastic strength, and shear yielding link may be developed when the supporting span reaching the shear yielding strength.

The inelastic behavior of the beam depends on the stiffness provided by the supporting member. For the practical consideration, the supporting member should be designed to remain in the elastic. Two types of the supporting member are presented in Fig. 2. Figure 2(a) shows the supporting member arranging inside the beam flanges. Channels are used in order to easily weld the member to the column. A box section supporting member which encloses the beam section is illustrated in Fig. 2(b).

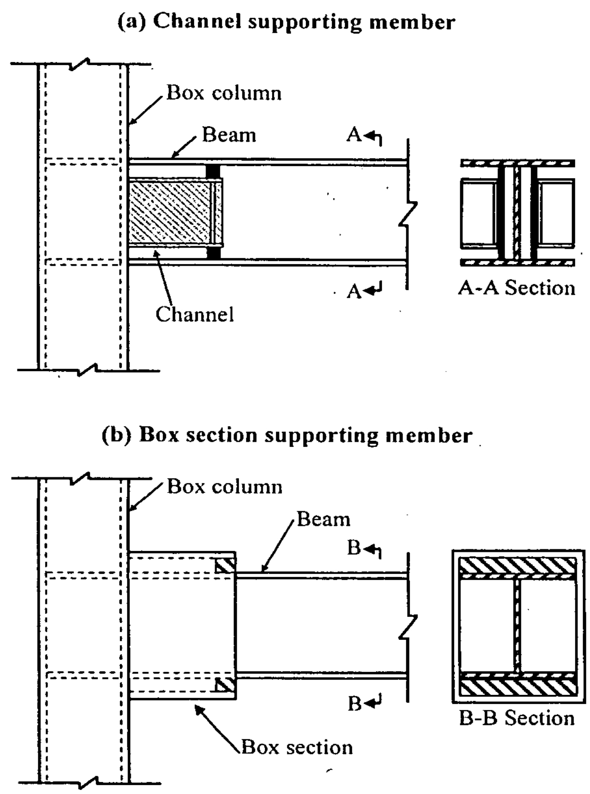


Fig. 2 Configurations of supporting members

EXPERIMENTAL PROGRAM

Test Specimens

Three specimens were designed in this study to verify the cyclic behavior and mode of failure. The beam-to-column subassemblages present an exterior joint, having an H-shaped H588×300×12×20 (mm) beam and a built-up box 550×550×27×27 (mm) column, and the steel being all ASTM A572 Grade 50. The length from loading point at the beam tip to the column face is 3600 mm, and the span between the column supports is 3000 mm. Designation of the specimens is tabulated in Table 1. Specimens CM600 and CM400 used channels as the supporting member, and intended to have flexural inelastic behavior on the beam at the supporting location. Figure 3 shows the connection details for specimen CM600. The channel used for specimen CM600 is 600 mm

long while that for specimen CM400 is 400 mm long. To prevent local failure of the beam section at the supporting location, steel band was designed to reinforce the beam flange. Moreover, a steel plate was welded to the channel to let the channel loaded through its shear center in order to prevent the channel from torsional deformation. As illustrated in Fig. 4, specimen BS600 utilized a box section supporting member, and shear yielding of the beam within the supporting span was expected. For specimen BS600, intermediate stiffeners were designed within the supporting span according to the design recommendation for the shear link.

Table 1 Designation of test specimens

Specimen	Beam and column	Supporting member	L_s (mm)	Yielding
CM600	H588×300×12×20 □550×550×27×27	C454×95×25×25	600	Flexure
BS600		□718×340×10×15	600	Shear
CM400		C454×85×15×15	400	Flexure

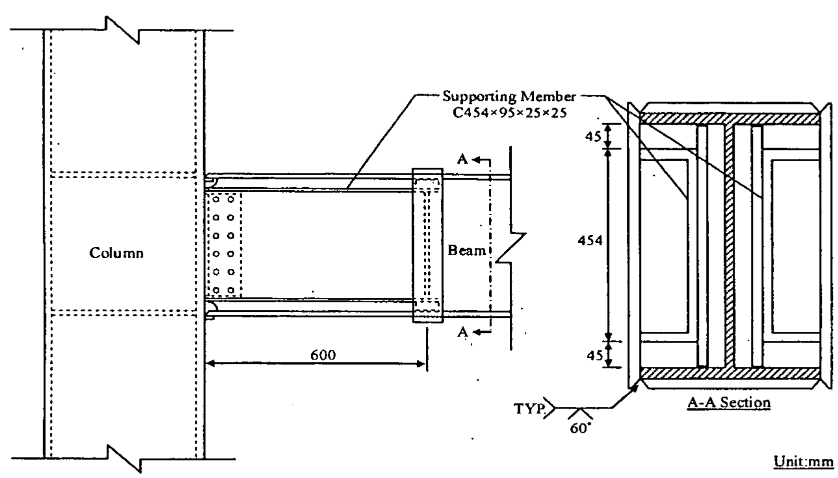


Fig. 3 Connection details of specimen CM600

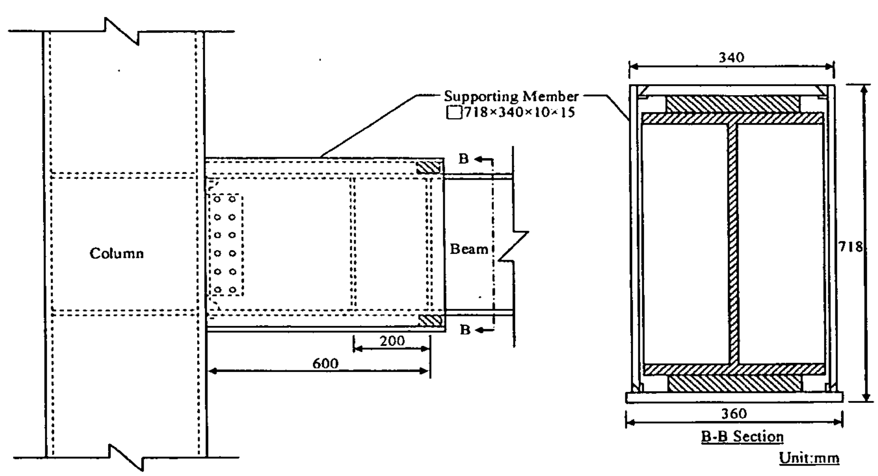
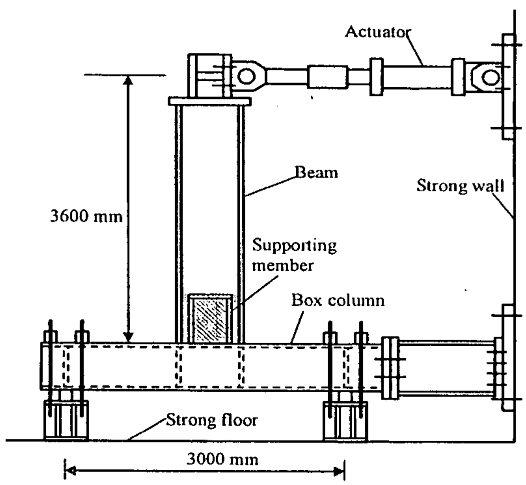


Fig. 4 Connection details of specimen BS600

Test Setup and Procedure

Figure 5 shows the test setup, representing the loading condition for the exterior joint, but the column was laid down horizontally. The beam tip was loaded by a hydraulic actuator which followed a predetermined loading history recommended by the AISC seismic provisions (AISC 2005). The loading sequence began with six cycles of ±0.375, ±0.5, and ±0.75% rad story drift angle, followed by four cycles of ±1% rad story drift angle and two cycles with amplitudes of over ±1.5% rad story drift angle until either the specimens failed or the actuator reached its stroke limitation. Lateral braces were installed to prevent possible out-of-plane deformation of the specimen.



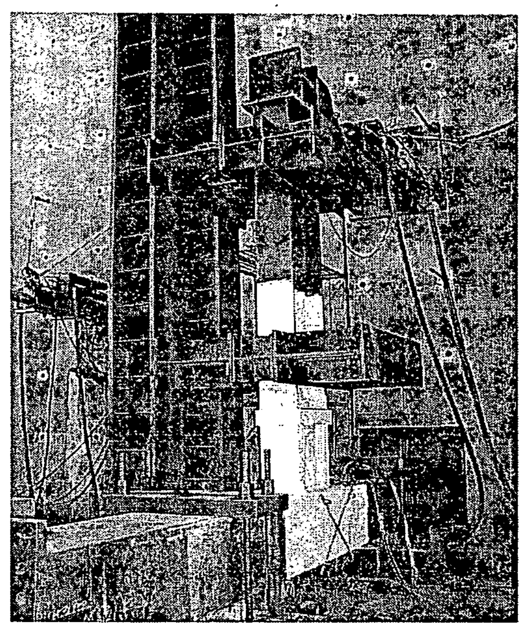


Fig. 5 Overall view of test setup

Observed Behavior

Similar behavior was observed from specimens CM600 and CM400. Whitewash on the beam flanges at the both sides of the supporting location initiated to flake, and the whitewash flaking continued to spread out when the excursion was increased. Notable local buckling was observed on the beam flange within the supporting span at the cycles of 4% rad story drift angle. For specimen CM600, due to the resistance of the supporting member, the buckling of the beam flange was restrained at 5% rad story drift angle, which caused no strength degradation. Figure 6 demonstrates the extensive yielding and local buckling of the beam flange within the supporting span for specimen CM600 at the cycles of 4% and 5% rad story drift angle. Notably, the whitewash on the supporting members did not flake at all. A similar observation was found in the testing of specimen CM400, but slight strength degradation was observed for specimen CM400 owing to the local buckling of the beam flange, as presented in Fig. 7. The supporting channels of both specimens did not show any sign of the yielding.

For specimen BS600, the progressive behavior of the beam could not be observed because of the enclosure of the box section supporting member. Whitewash flaking was noticed at the supporting location at 0.75% rad story drift angle. Due to the concentrated load at the supporting location, locally excessive deformation occurred on the flange of the box section supporting member, and resulted in a gap between the beam flange and the supporting member after 3% rad story drift angle. In such a case, the supporting member did not provide the intended reactive load to the beam, and then the beam acted without withstanding from the supporting member. At the second cycle of the +4% rad story drift angle, specimen BS600 failed and caused the load dropping suddenly. Testing was continued toward the reverse excursion, and specimen BS600 completed the cycle of -5% rad story drift angle without showing any damage. After the conclusion of the test, the box section supporting member was removed to examine the failure of the beam. As shown in Fig. 8, shear yielding of the beam web

within the supporting span was noticed, and the fracturing of the beam bottom flange at the fusion zone of the beam flange complete joint penetration weld was also presented.

(a) Yielding pattern at 4% rad story drift **(b) Local buckling at 5% rad story drift**

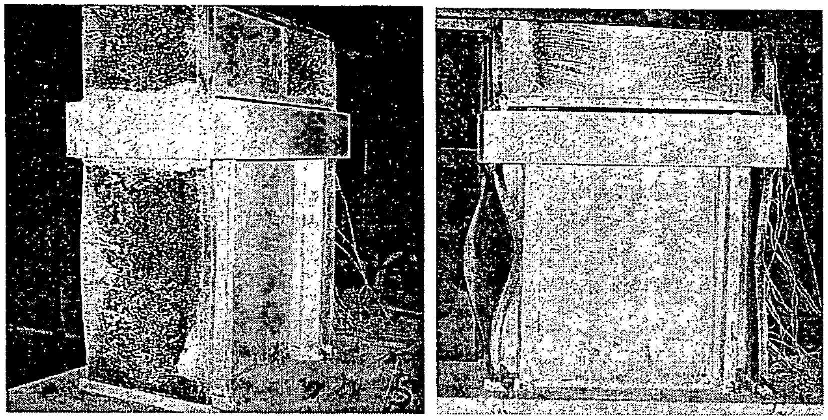


Fig. 6 Yielding and local buckling of specimen CM600

(a) Yielding pattern at 4% rad story drift **(b) Local buckling at 5% rad story drift**

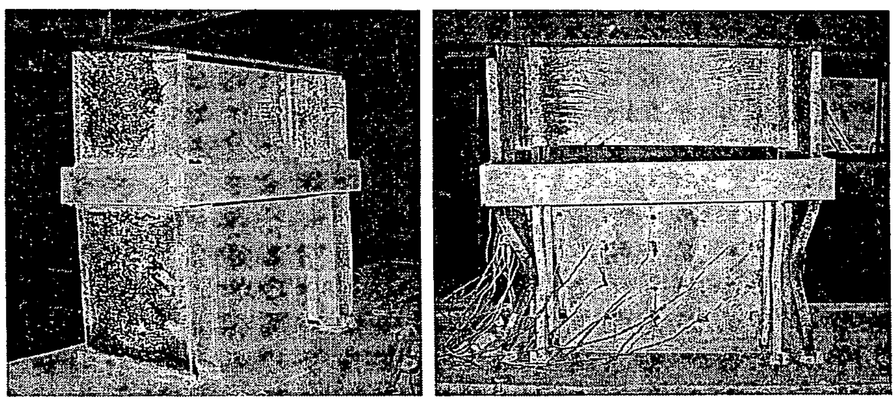


Fig. 7 Yielding and local buckling of specimen CM400

(a) Fracturing of beam bottom flange **(b) Shear yielding on the beam web**

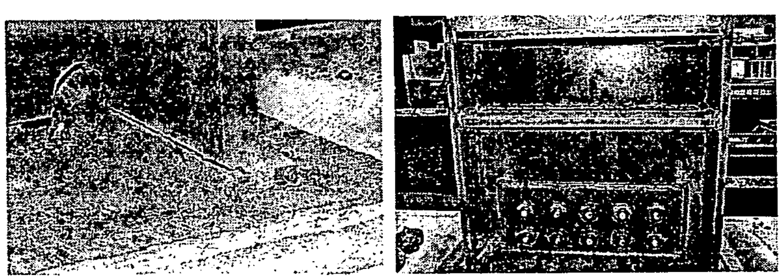
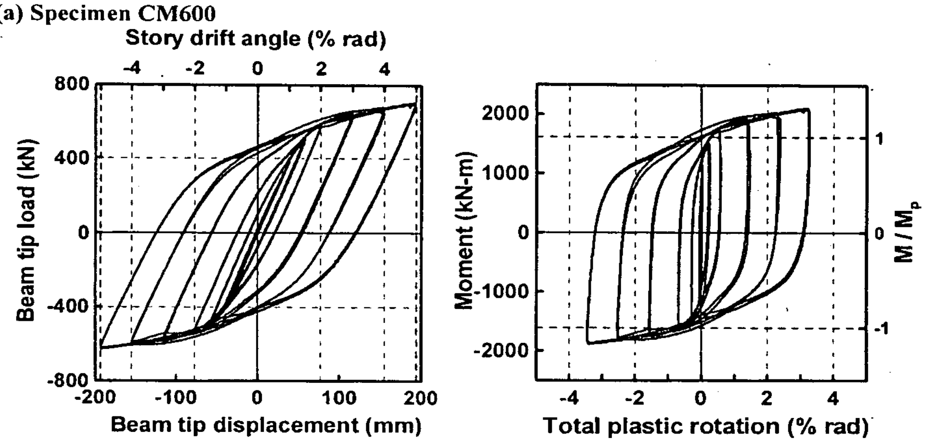


Fig. 8 Specimen BS600 at conclusion of test after removal of the box supporting member

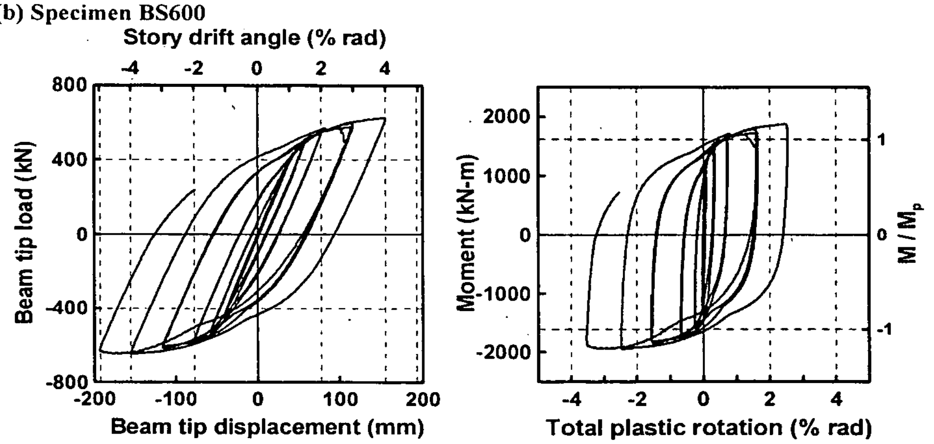
Hysteresis loops

The hysteresis loops for the three specimens are presented in Fig. 9. Both beam tip load-displacement curves and moment-total plastic rotation curves are indicated. The moment was calculated by multiplying the beam tip load by the distance from the beam tip to the supporting location. The total plastic rotation was calculated by subtracting the elastic rotation from the total rotation of the specimen. As shown in Fig. 9(a) and 9(c), satisfactory hysteresis curves can be obtained from specimens CM600 and CM400, with minor strength degradation caused by the local buckling of the beam. Specimens CM600 and CM400 developed +3.27%, -3.45% and +3.44%, -3.52% rad plastic rotations, respectively, which satisfy the requirement for moment connection used in the special moment frames (AISC 2005). However, specimen BS600 reached +2.54%, -3.54% rad plastic rotations because of the unexpected local failure of the box supporting member, resulting in the fracture of the beam bottom flange at the beam-to-column joint.

(a) Specimen CM600



(b) Specimen BS600



(c) Specimen CM400

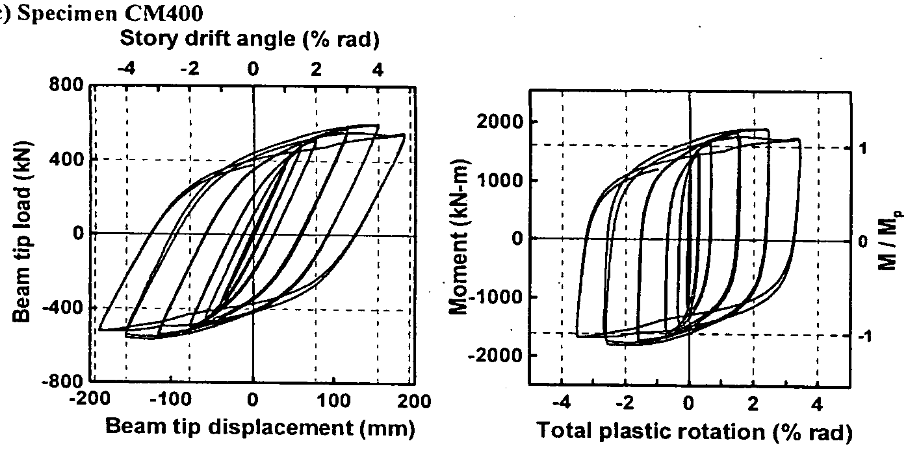


Fig. 9 Hysteresis responses of specimens

CONCLUSION

The supporting member used in beam-to-column connections is intended to generate a reactive load to the beam, and to reduce the moment demand at the beam-column interface. Depending on the stiffness provided by the supporting member, the beam may be either flexural yielding at the supporting location or shear yielding within the supporting span. Three full-scale beam-column subassemblies with the supporting member have been tested to verify their cyclic behavior. The tests have shown that connections with the supporting member can achieve stable hysteresis behavior under large story drift angle by either forming a plastic hinge at the supporting location or developing a shear yielding on the beam web within the supporting span. Although satisfactory cyclic behavior can be developed by adding the supporting member, constructional feasibility needs to be overcome.

ACKNOWLEDGEMENTS

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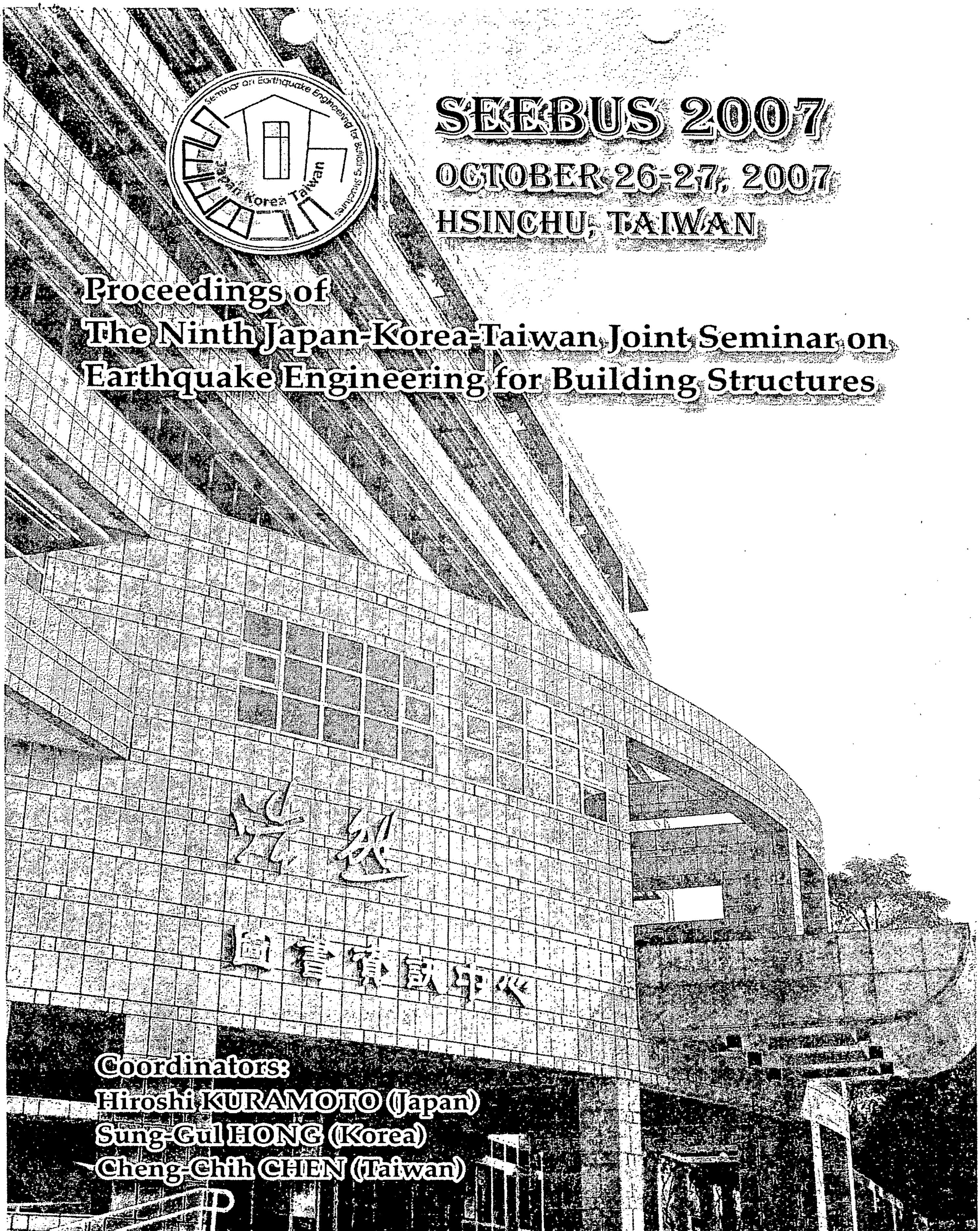

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